Bull. ERS, No. 31 (1998)

Seismic Response Analysis of the Higashi-Kobe Bridge in the 1995 Hyogoken-Nanbu Earthquake

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SUMMARY

This paper presents results of the observation and response analysis of the Higashi-Kobe cable-stayed bridge during the Hyogoken-Nanbu Earthquake of January 17, 1995. With three-dimensional finite element models, the response of the bridge is simulated for transverse and longitudinal input motions. A comparison among two small earthquakes and the Hyogoken-Nanbu Earthquake is made.

1. INTRODUCTION

At 5:46 a.m. on January 17, 1995, Kobe was struck by the most devastating earthquake[1] in Japan since the Great Kanto Earthquake of 1923. The epicenter was located at 34° 36"N and 135° 00"E having JMA magnitude 7.2, surface wave magnitude (M_S) 7.2, moment magnitude (M_W) 6.9 and focal depth 14 km. The earthquake was a very harsh test for most of the structures.

The Higashi-Kobe Bridge is a part of the Osaka Bay Route, of the Hanshin Expressway. This route is an 80 kilometers expressway stretching from the western end of Kobe to the southern end of Osaka (Fig. 1). The Higashi-Kobe Bridge spans the Higashi-Kobe Channel connecting two reclaimed land areas. The channel is 500 wide and has a 455m seaway, where large ferries frequently pass to and from the nearby Ohgi Ferry Terminal. Having a long natural period (approximately 4.4 seconds) the resonance phenomenon is unlikely to happen to the bridge. However the Hyogoken-Nanbu Earthquake affected also this bridge.

In the previous research[2], it was determined that interaction between the foundation and the supporting soil plays a decisive role for the earthquake response, especially for large structures like cable-stayed bridges. Thus with a finite element model, a simulation study was performed for the longitudinal direction excitation of the bridge. In this paper, the linear dynamic response from two small earthquakes and nonlinear dynamic response from the Hyogoken-Nanbu Earthquake in both the longitudinal and transverse directions are examined using three-dimensional finite element codes.

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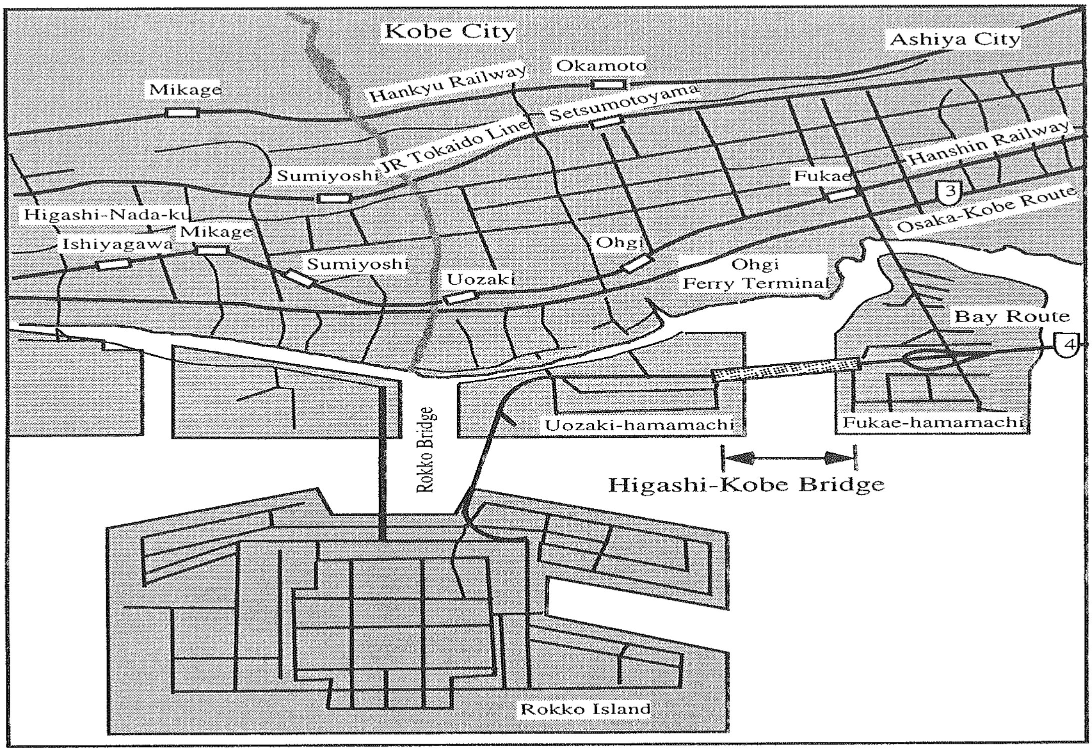


Fig. 1 Location of the Higashi-Kobe Cable-Stayed Bridge

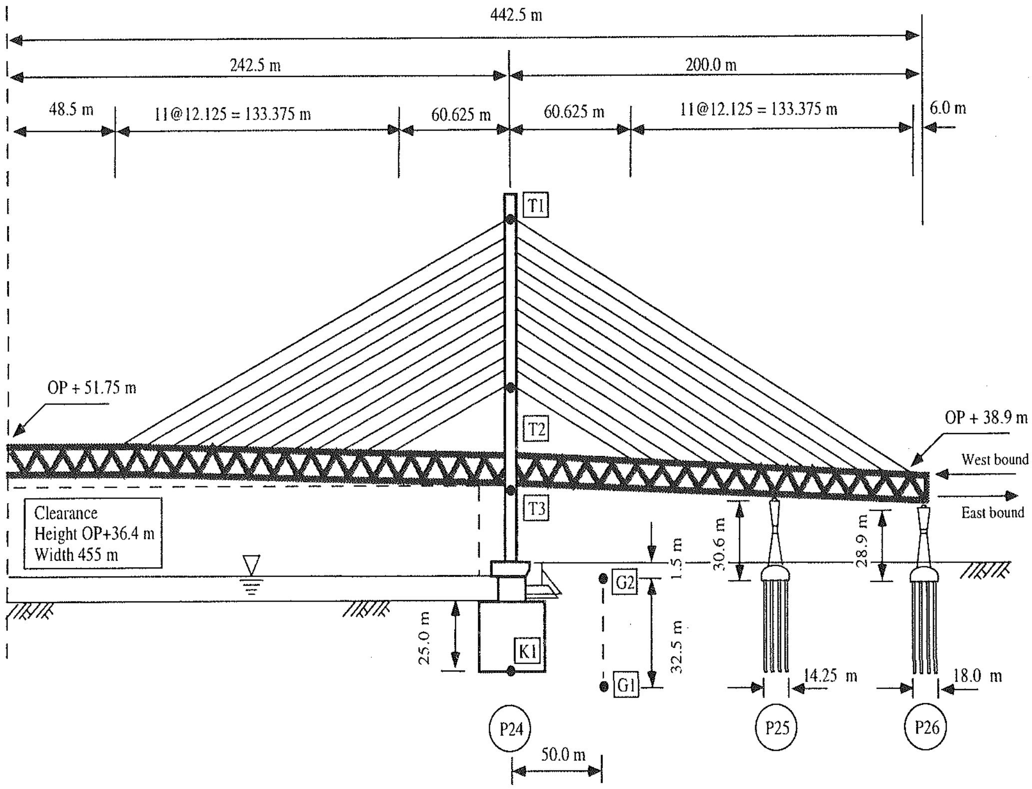


Fig. 2 Scheme of the bridge and locations of the seismometers

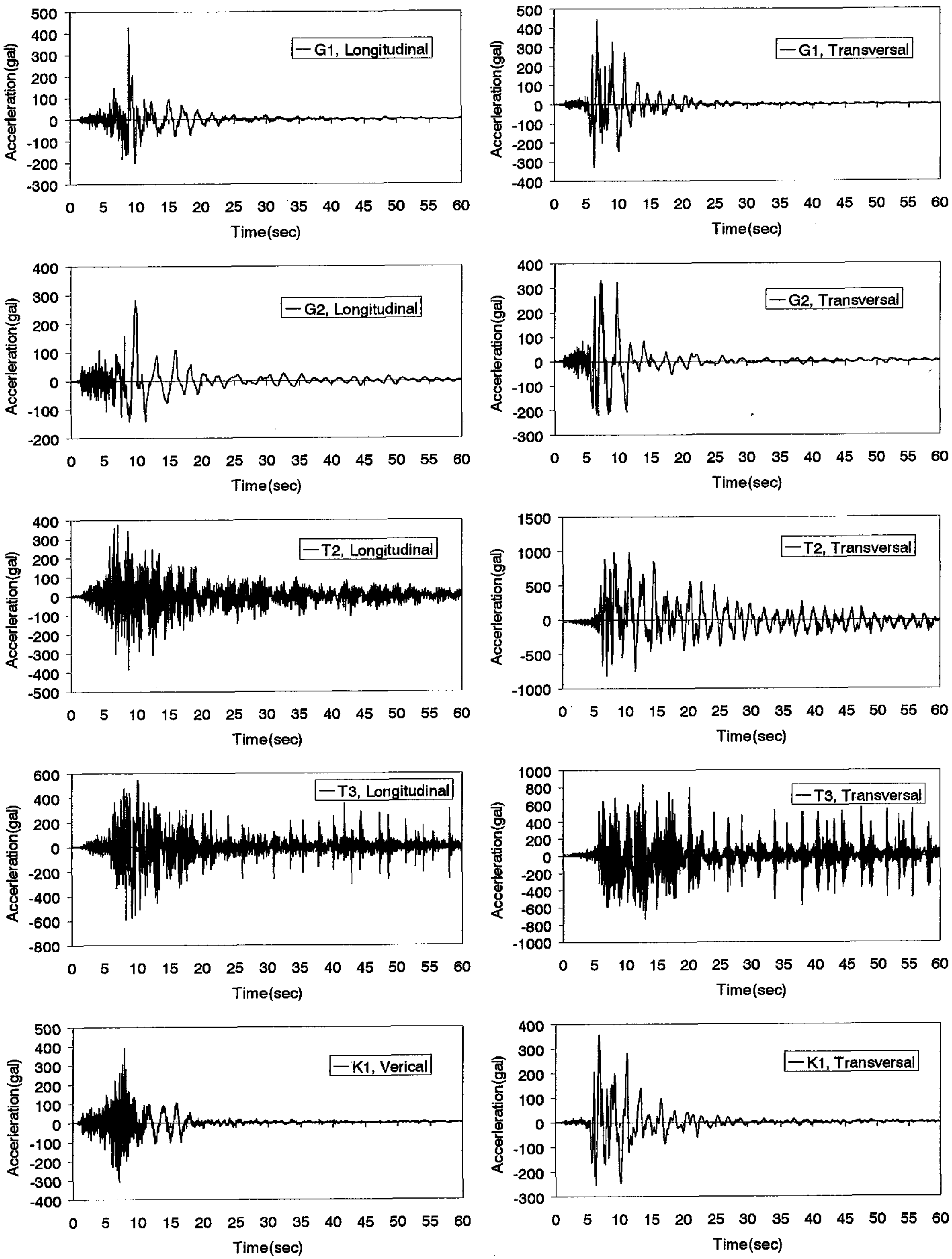


Fig. 3 Acceleration time histories in the 1995 Hyogoken Nambu Earthquake

2. DESCRIPTION OF THE HIGASHI-KOBE BRIDGE

The Higashi-Kobe Bridge was designed[3] by the Hanshin Expressway Public Corporation. The consultant services of Sogo Engineering Inc., Osaka, were employed to perform seismic analysis.

The Higashi-Kobe Bridge is a three-span cable-stayed bridge 885m long, and is one of the longest cable-stayed bridges in the world. The bridge spans the Higashi-Kobe Channel between two areas of reclaimed land (Fig. 1). It is a double-deck bridge with the upper and lower roadways having three lanes each.

Figure 2 presents a scheme of the eastern half of the bridge. The bridge piers at the east side are named P24, P25 and P26. Accelerometers are placed on three locations on the tower at P24 and are named T1, T2 and T3 in Fig. 2. The soil response during earthquakes is recorded by accelerometers buried in the ground at depths 34 (G1 as in Fig. 2) and 1.5 m (G2 as in Fig. 2) at a 50m distance from the foundation at P24. An accelerometer is also installed at the bottom of the caisson (K1 in Fig. 2).

The center span of the bridge is 485 long, and each side span measures 200m. Because the center span is rather long, a pendulum pier is used for each of the side spans to reduce the middle span deflection. The bridge has a truss girder and employs a double-sided multi-cable system with 96 stay cables in total. Each cable consists of 241 to 301 wires of 7mm diameter. The cable is clad in polyethylene tube for corrosion protection.

The unique feature of the bridge is that the main girder can move longitudinally on all supports, with the corresponding displacements restricted mainly by the cables. This supporting system was adopted with the aim of lengthening the fundamental period of the bridge. This long natural period led to a reduction in the seismic induced forces, and consequently in the size of the towers and foundations. The truss floor system uses a steel deck, reducing the weight of the superstructure. The floor system, composed with the upper and the lower chords, serves as the main girder.

Each H-shaped tower consists of two vertical columns tied by upper and lower horizontal beams. The columns stand 146.5 m high and 24 m apart. The columns are constructed using hollow steel elements with rectangular cross section joined by welding, except five locations where high strength bolts are used to correct uneven shrinkage caused by welding.

The tower foundations are pneumatic caissons of 35(W) x 32(H) x 26.5(D) m. The inside of each caisson is divided into 6 rows of 6 cells with partition walls. The steel shells of the caisson have been connected with the reinforcing bars and used as mold for the concrete foundation. The secondary piers at the side spans are founded on piles. Some additional remarks are in Table 1.

3. RESPONSE DURING THE 1995 HYOGOKEN-NANBU EARTHQUAKE AND THE SMALL EARTHQUAKES

Figure 3 shows response acceleration time histories of the Higashi-Kobe Bridge and the surrounding soil. The records at the top of the tower (T1 as in Fig. 2) are clipped. In Table 2, the maximum accelerations, velocities and displacements are shown at each observation point. The records of the downhole accelerometer G1 (GL-34.0m as in Fig.

Table 1. Specifications of the Higashi-Kobe Bridge

Type	Three-span continuous steel cable-stayed bridge
Road category	Group2, Class 1
Design velocity	80 km/h
Roadway	2decks x 3lanes
Length	200+485+200=885 m
Width	13.5m x 2 decks
Tower	H-shaped tower (146.5 m)
Main girder	Warren truss (height 9 m)
Cables	Harp pattern multi cable (12 cables in a plane)
Substructure	Caisson foundation (for towers) Pile foundation (for piers)
Weight of the superstructure	Main girder 141,000 KN Towers 79,000 KN Cables 13,000 KN Piers 17,000 KN Others 24,000 KN Total: 274,000 KN
Specification of the caissons	Weight of the steel shells 9,500 x 2=19,000 KN Volume of the concrete 15,300 x 2=30,600 m^3 Weight of the reinforcing bars 13,000 x 2=26,000 KN

Table 2. Maximum recorded response to the 1995 Hyogoken-Nanbu Earthquake

Name and Position	Orientation	Acceleration [cm/s^2]	Velocity [cm/s]	Displacement [cm]
G1 (GL-34m)	Longitudinal	425.4	71.2	27.8
	Transverse	443.4	76.0	34.3
G2 (GL-1.5m)	Longitudinal	228.0	84.5	51.2
	Transverse	325.8	90.7	49.5
	Vertical	295.8	35.0	14.9
K1 (Bottom of Foundation at P24)	Longitudinal	333.9	77.5	34.0
	Transverse	354.9	79.1	39.4
	Vertical	389.3	34.1	12.6
T2 (Middle of Tower at P24)	Longitudinal	385.7	29.1	18.7
	Transverse	1000*	225.1**	117.6**
T3 (Tower at P24, level of main girder)	Longitudinal	596.3	90.7	33.9
	Transverse	806.5	105.7	51.3
	Vertical	806.7	71.1	37.8

Notes: (*)- Overscaled gauge, (**)- The value is calculated from an overscaled record

2) shows that the near fault ground motions include large pulses with long period, which are potentially damaging to structure with long natural period such as long bridges.

The time history of acceleration at the main girder level (T3, as in Fig. 2) shows multiple pulses that decrease towards the end of the record. This phenomenon was suspected to be caused also by the relaxation of the cables. Poundings and damaged supports may cause a similar effect. By performing dynamic response analysis by SASSI[4] and a static analysis by NASTRAN[5] with three-dimensional models was found that several cables were relaxed during the earthquake. A discussion about this problem will be later in the paper.

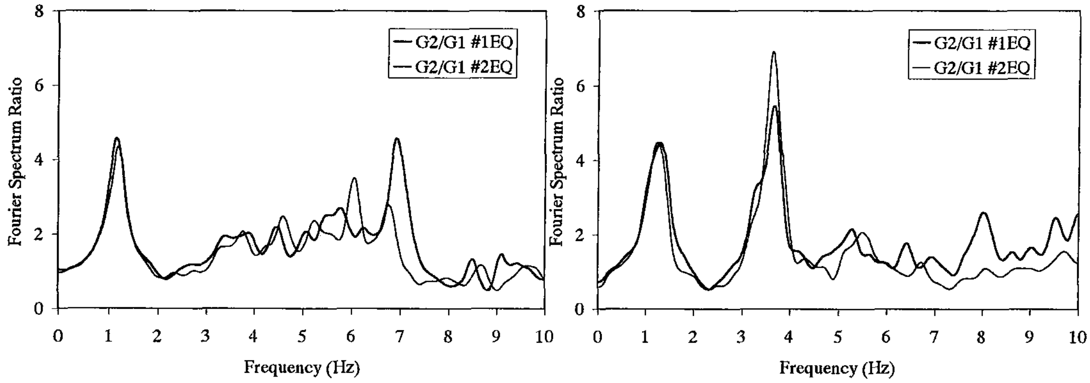
Comparing the time histories from downhole accelerometer G1 (GL-34.0m as in Fig. 2) and the surface accelerometer G2 (GL-1.5m), it can be seen that the acceleration at the surface is smaller than the one at 34m depth and shows longer period. These observations suggest that the surface soil layers were liquefied during the earthquake. This hypothesis was confirmed numerically by the previous research[2].

Because the Higashi-Kobe Bridge is a very complex structure it is useful to compare the response during the Hyogoken-Nanbu Earthquake with the response during small earthquakes. The characteristics of the small earthquake (#1EQ) of January 25, 1996 are summarized in Table 3. The second small earthquake is named #2EQ, and had a peak ground acceleration of 25 cm/s/s at G2.

Figure 4 compares Fourier spectrum ratios between the surface and the downhole records, evaluated from the #1EQand #2EQ. One can see that around 1 Hz there is a peak in both the longitudinal and transverse directions. Also can be seen that for the higher range of frequencies the response of soil differs in the two discussed directions.

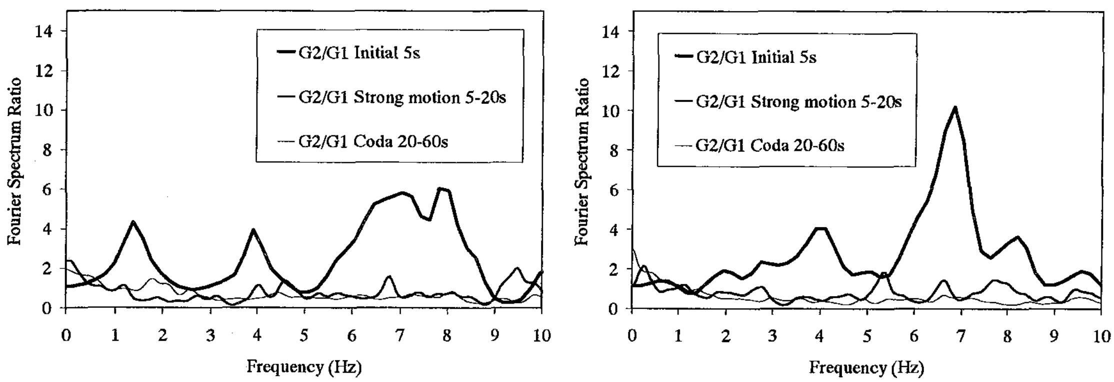
Figure 5 shows Fourier spectrum ratios between the downhole (GL-34m) and surface (GL-1.5m) records. It is observed that in the first 5 seconds amplification is larger than in the next 20 seconds of the earthquake. In the last 40 seconds there is no significant amplification. This can be explained with the occurrence of liquefaction, which suppresses the energy transfer. Also, if one compare the longitudinal and transverse directions it can be said that the Fourier amplitude ratios in the two directions differ significantly in terms of both frequency and amplitude. Although the response of the soil was supposed to give similar results in both the transverse and longitudinal directions this was not true. This may be caused by the fact that the soil properties differ for the two directions. In the longitudinal direction on one side of the caisson, there is water down to about 13m depth and then soil while in the transverse direction, both sides of the caisson are continued by soil.

Figure 6 compares Fourier spectrum ratio between the free field at a depth of 1.5m and the mid-height of the bridge tower, evaluated from the record of the Hyogoken-Nanbu Earthquake and from the small event No.1EQ. It is known that for short rigid structures, such as reinforced concrete buildings, the predominant frequencies decrease as the excitation increase. For this long-period structure, the decrease in the predominant frequency was not observed.



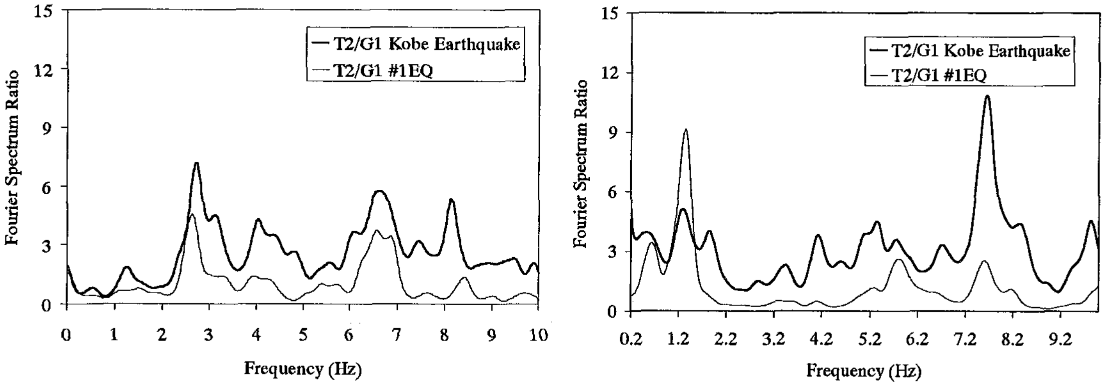
(a) Longitudinal (b) Transverse

Fig. 4 Comparison of Fourier spectrum ratios of the two small earthquakes



(a) Longitudinal (b) Transverse

Fig.5 Fourier spectrum ratios between surface and downhole acceleration records of the Hyogoken-Nanbu Earthquake



(a) Longitudinal (b) Transverse

Fig. 6 Comparison of Fourier spectrum ratios evaluated from large and small earthquakes

Table 3. Maximum recorded response to a small earthquake of January 1996.

Name and Position	Orientation	Acceleration [cm/s²]	Velocity [cm/s]	Displacement [cm]
G1 (GL-34m)	Longitudinal	25.0	1.5	0.13
	Transverse	23.0	1.5	0.07
G2 (GL-1.5m)	Longitudinal	42.0	2.6	0.26
	Transverse	31.0	1.6	0.13
	Vertical	24.0	1.0	0.07
K1 (Bottom of Foundation at P24)	Longitudinal	15.0	1.5	0.15
	Transverse	13.0	1.0	0.08
	Vertical	13.0	0.9	0.04
T2 (Middle of Tower at P24)	Longitudinal	31.0	1.5	0.09
	Transverse	11.0	1.6	0.14
T3 (Tower at P24, level of main girder)	Longitudinal	66.0	3.8	0.32
	Transverse	65.0	1.9	0.13
	Vertical	19.0	0.9	0.05

Table 4. Soil properties used for dynamic analysis

	Small earthquake				The Hyogoken-Nanbu Earthquake			
Soil type	Shear wave velocity Vs (m/s)	Poisson ratio ν	Unit weight γ kN/m³	Damping ratio D (%)	Shear wave velocity Vs (m/s)	Poisson ratio ν	Unit weight γ kN/m³	Damping ratio D (%)
1	151	0.48	18	3.3	20	0.48	18	12
2	144	0.48	18	5.6	30	0.48	18	12
3	271	0.44	18	3.6	40	0.44	18	12
4	269	0.44	18	4.0	40	0.44	18	12
5	107	0.49	16	5.3	70	0.49	16	10
5a	107	0.49	15	5.3	70	0.49	15	8
6	116	0.49	16	3.8	50	0.49	16	10
6a	116	0.49	15	3.8	50	0.49	15	10
7	302	0.48	19	3.2	234	0.48	19	5
8	363	0.47	19.5	2.8	297	0.47	19.5	3.3
9	238	0.49	19.5	0.4	169	0.49	19.5	3
10	331	0.48	19.5	3.0	317	0.48	19.5	7.1
11	207	0.49	19.5	0.7	198	0.49	19.5	7.1
12	410	0.49	19.5	0.2	380	0.49	19.5	7.1

4. DESIGN ACCELERATION RESPONSE SPECTRA

The dynamic response of the bridge to earthquakes was evaluated using the response spectrum approach[3]. To make this response spectrum, first some observed seismic waves were selected that were appropriate as analysis material. Once the design spectrum was obtained, it was assumed that the tower bases were subjected to those seismic waves.

The design spectrum (as in Fig. 7) was provided with a relatively large safety margin in the long period range. This is because the Higashi-Kobe Bridge has an unprecedented long natural period of longitudinal sway mode oscillation. To make this response spectrum, 1,000m deep bedrock at the construction site was considered. This depth is somewhat deeper than usual to increase the safety margin. The maximum acceleration of seismic waves at the bedrock was assumed to be 160 cm/s/s as an expected value during a 100-years return period. The design spectrum was determined by taking an envelope curve of mean spectrum of three seismic waves: at the bedrock, at the 80m deep design ground base, and at the tower base.

Figure 7 shows a comparison among design acceleration spectrum and the response spectra evaluated from the records at G2 and G1 (GL-1.5m and GL-34m, respectively, as in Fig. 2). From the comparison it resulted that the response spectra surpass the design one in ranges like 0.1 to 1 and 2 to 3 seconds period.

6. SIMULATION OF THE SEISMIC RESPONSE

6.1 Seismic Response during the Small Earthquake

The response analysis was conducted using the SASSI three-dimensional model. This finite element model is able to take into account the effects of soil-structure interaction by discretization of the soil surrounding the foundations (Fig. 8 and Table 4). For simplicity, it was assumed that the bridge model is symmetrical about two planes of the bridge – longitudinal and transverse. Thus a quarter of the model was used.

Figure 9 (a) shows the comparison of Fourier spectrum ratios between downhole (GL-34.0m) and surface (GL-1.5m) for the small earthquake (#1EQ) evaluated from records and calculations in the longitudinal direction. There can be observed that the main peak in Fourier spectrum ratio is well simulated by the model. The recorded is slightly higher than the calculated one, but shows no shift in the predominant frequency. Figure 9 (b) compares Fourier spectrum ratio between downhole (GL-34.0m) and the mid-height of the bridge tower, evaluated from the records and calculations for the same small earthquake in the longitudinal direction. In this comparison one can see that the model is able to simulate the response of the bridge during the small earthquake. The main peak, situated in the 2-3Hz range, is almost coincident for the recorded and calculated motions.

Figure 10 (a) shows the Fourier spectrum ratios between top of the tower (T1) and main girder level (T3) for the transverse direction, evaluated from records and calculations for the #1EQevent. Figure 10 (b) shows the ratio between the middle of the tower (T2) and the bottom of caisson (K1). The simulation is not as good as for the

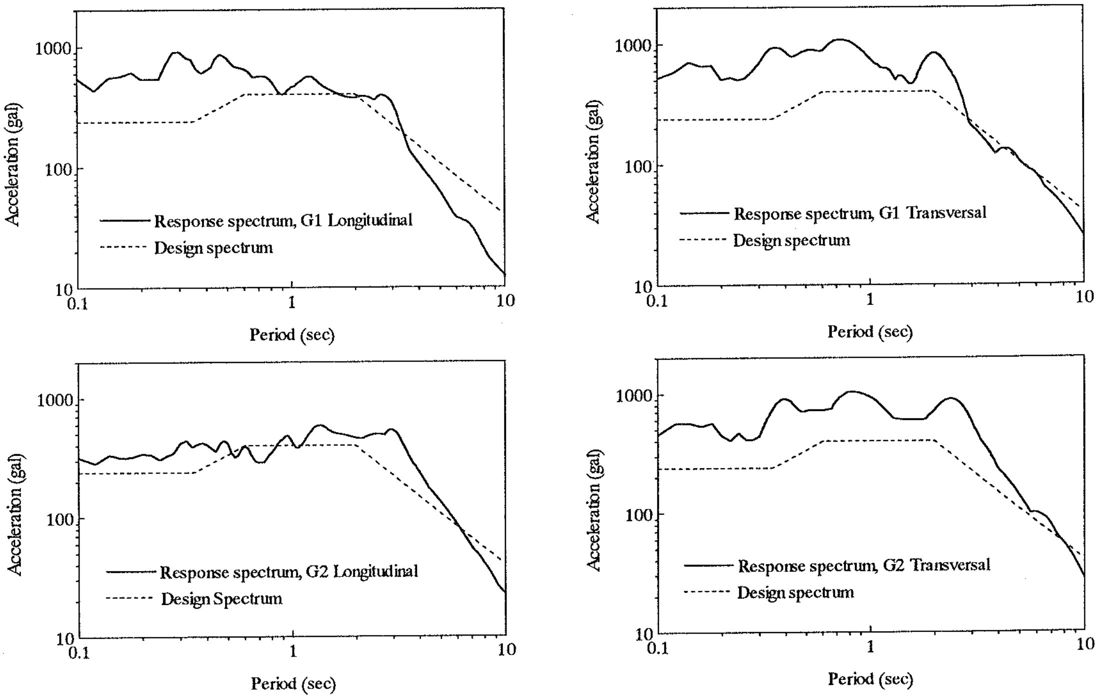


Fig.7 Comparison between the design spectrum of the Higashi-Kobe Bridge and the acceleration response spectrum evaluated from Hyokoken-Nanbu Earthquake

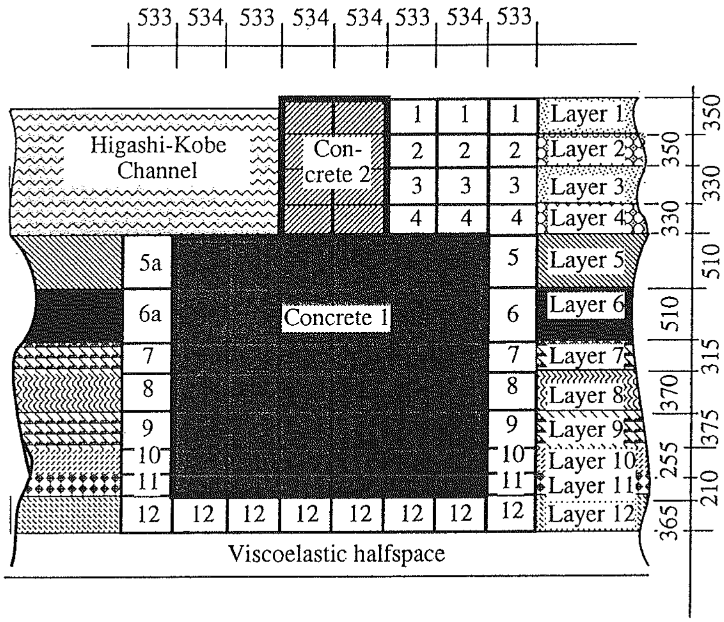
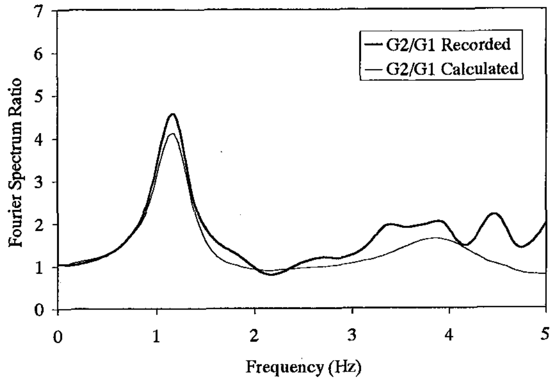
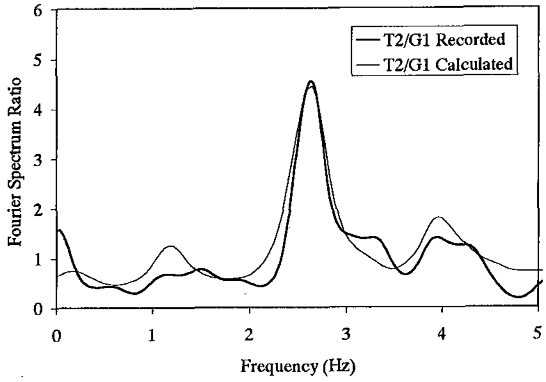


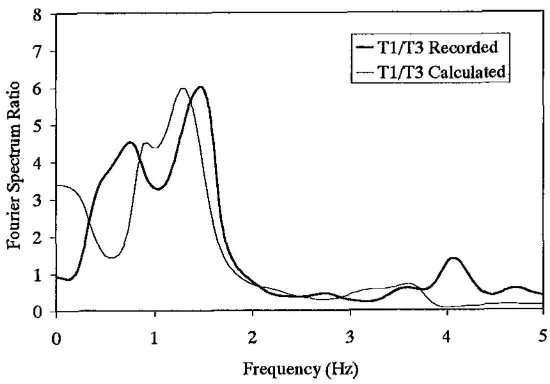
Fig. 8 Soil discretization used in SASSI

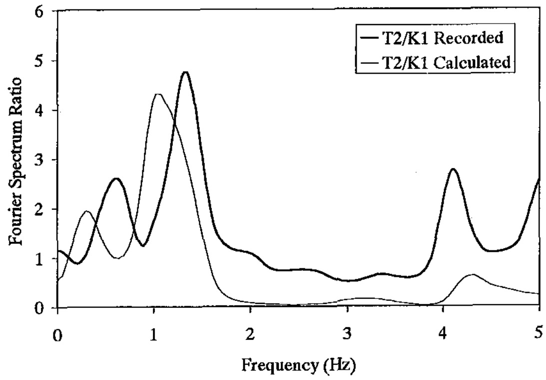




(a) (b)

Fig.9 Simulation of the response during the No.1EQ (longitudinal direction)





(a) (b)

Fig. 10 Simulation of the response during the No.1EQ (transverse direction)

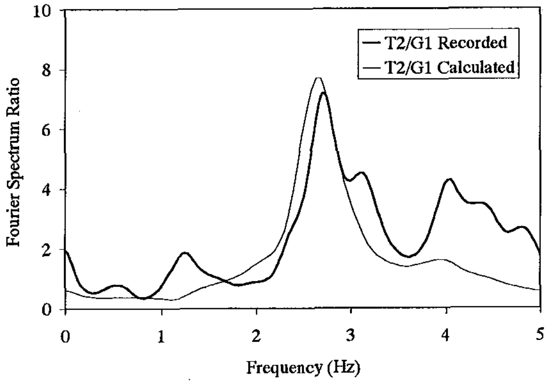


Fig. 11 Simulation of the response during the 1995 Hyogoken-Nanbu Earthquake (longitudinal direction)

longitudinal direction. This shows that the model should be improved to simulate the response of the bridge in the transverse direction more accurately.

5.2 Seismic Response during the Hyogoken-Nanbu Earthquake

Figure 11 compares also Fourier spectrum ratio between the downhole (GL-34.0m) and the mid-height of the bridge tower, evaluated from the records and calculations for the Hyogoken-Nanbu Earthquake in the longitudinal direction.

In this case, the model simulates the overall behavior of the system. One can see that the model cannot simulate very well the nonlinear response of the soil and structure system by equivalent linear soil parameters[2].

7. EXAMINATION ON NONLINEARITY

7.1 Cable Relaxation

The SASSI model description can be found in reference [2]. The MSC/NASTRAN model is shown in Figure 12. The MSC/NASTRAN model uses four types of elements: beam, rod, mass and spring. The beam elements were used to model the main girder and the towers. The rod elements, which can carry only tensile force, were used for cables. The mass elements were used for the concentrated masses and the spring elements were used for the pile and the caisson foundations.

From the static gravity analysis with the three-dimensional NASTRAN model, axial forces in cables were obtained. From the dynamic analysis with the three-dimensional SASSI model with the excitation in the longitudinal direction of the bridge, the maximum forces in cables were obtained. A comparison was made between the forces in the cables obtain from static analysis and the maximum forces obtained from the seismic analysis and it resulted that several cables were relaxed during the Hyogoken-Nanbu Earthquake (Fig. 13 and Table 5).

Hence, it can be concluded that the nonlinear dynamic analysis considering cable relaxation should be conducted, to simulate the behavior of the cables in a more realistic manner.

7.2 Pulse-Like Response Acceleration Time Histories

Comparing the time histories of acceleration from T2 (middle of the tower level), one can say that in the longitudinal direction the response acceleration is smaller than in the transverse direction. This may be explained by the fact that the first longitudinal natural period of the bridge is about 4.8 seconds, while the first transverse mode has a period of about 2.5 seconds[3]. Thus, the amplification for the transverse direction may be caused since the input motion had more power around the first transverse mode. Figure 14 shows the Fourier spectra of T3 records from the 1995 Hyogoken-Nanbu Earthquake and #1EQaftershock. It can be seen that in the transverse direction there is amplification in the higher frequency range. This fact is not observed for the longitudinal direction. This transverse amplification may be the cause of the beating pattern observed in the T3

Nodes	348
Elements	568
Materials	2
Properties	96
Element types	4

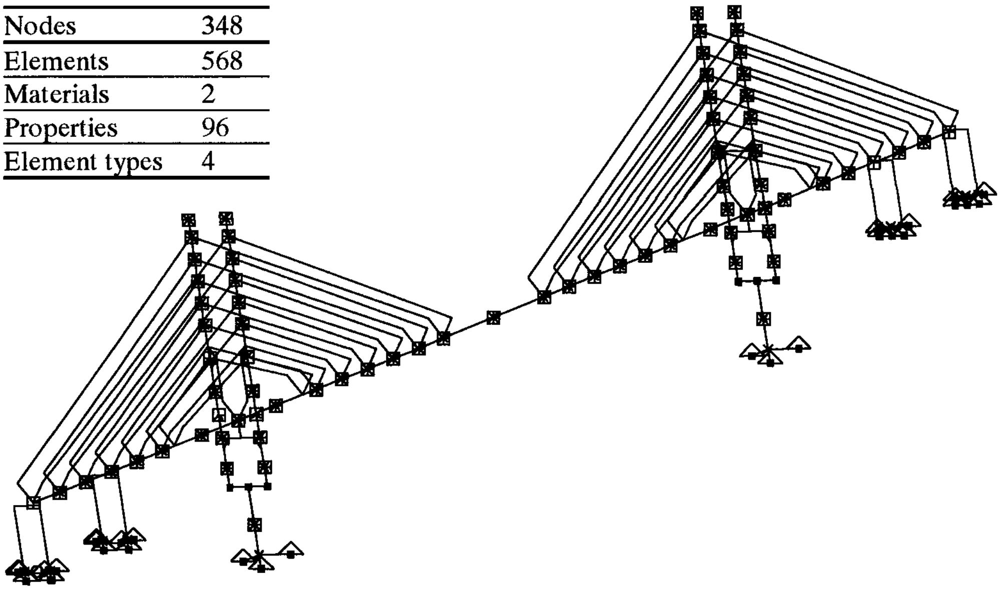


Fig. 12 Three dimensional MSC/NASTRAN finite element model

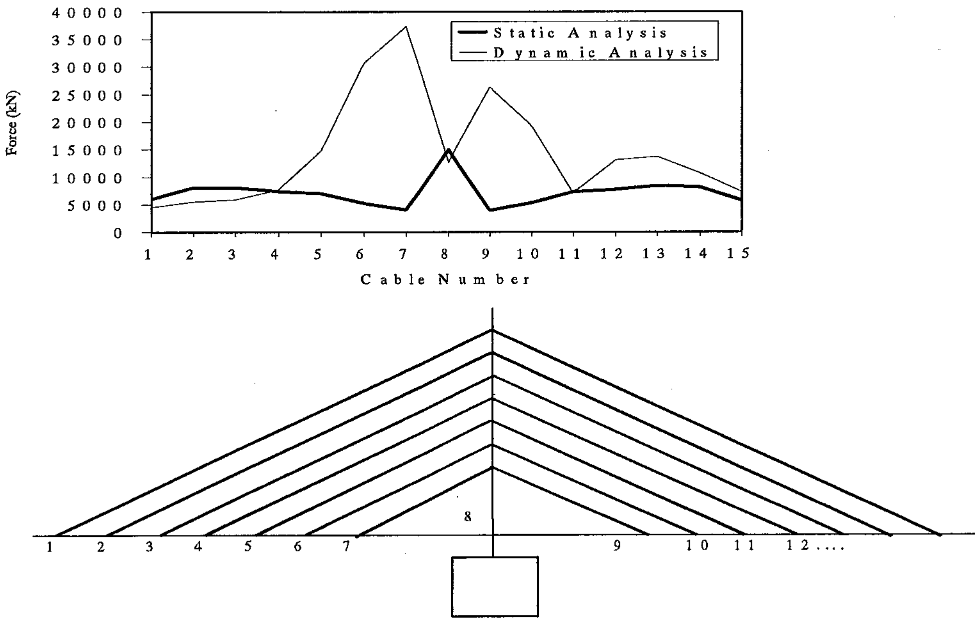


Fig. 13 Forces in cables and cable arrangement

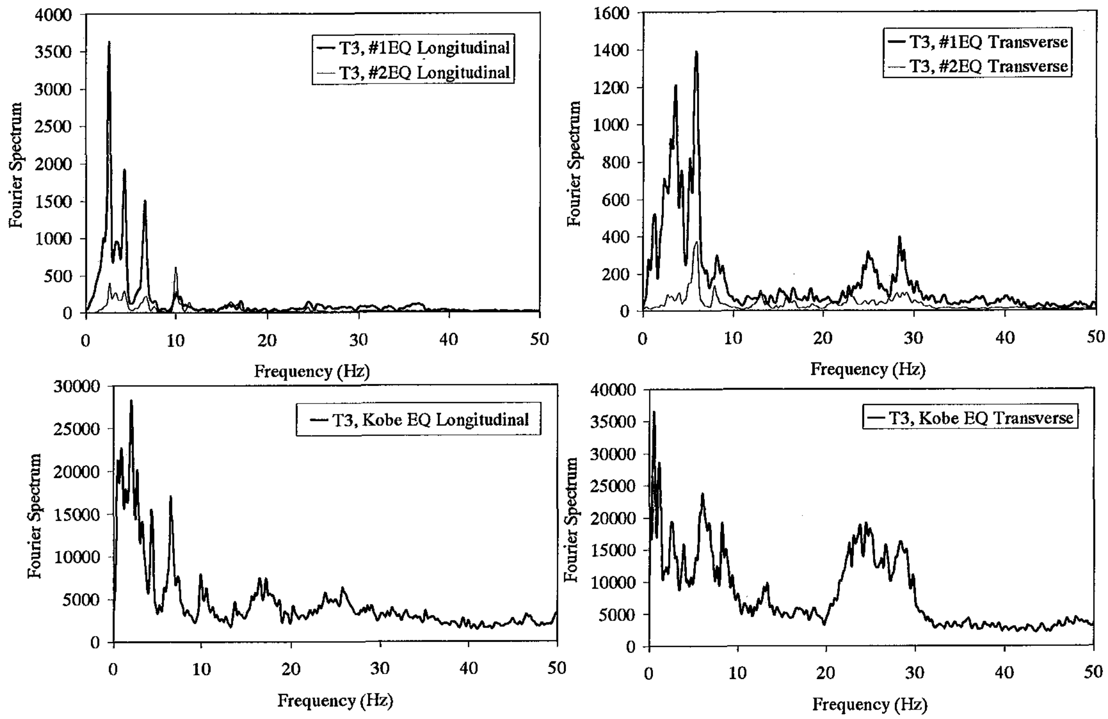


Fig. 14 Comparison of Fourier spectra evaluated from #1EQ, #2EQ and The Hyogoken-Nanbu Earthquake

Table 5. Forces in cables from analyses

Cable	Axial force in cable from NASTRAN due to gravity (KN)	Maximum axial force in cable from SASSI due to earthquake (KN)
1	-6012	+4459
2	-8023	+5461
3	-8045	+5909
4*	-7336	+7709
5*	-6997	+14740
6*	-5173	+30640
7*	-3935	+37220
8	-14890	+12580
9*	-3921	+26260
10*	-5263	+19210
11*	-7245	+7092
12	-7641	+13000
13*	-8375	+13630
14*	-8122	+10620
15*	-5668	+7262

Note: (*)-Cable that was relaxed during the Hyogoken-Nanbu EQ.

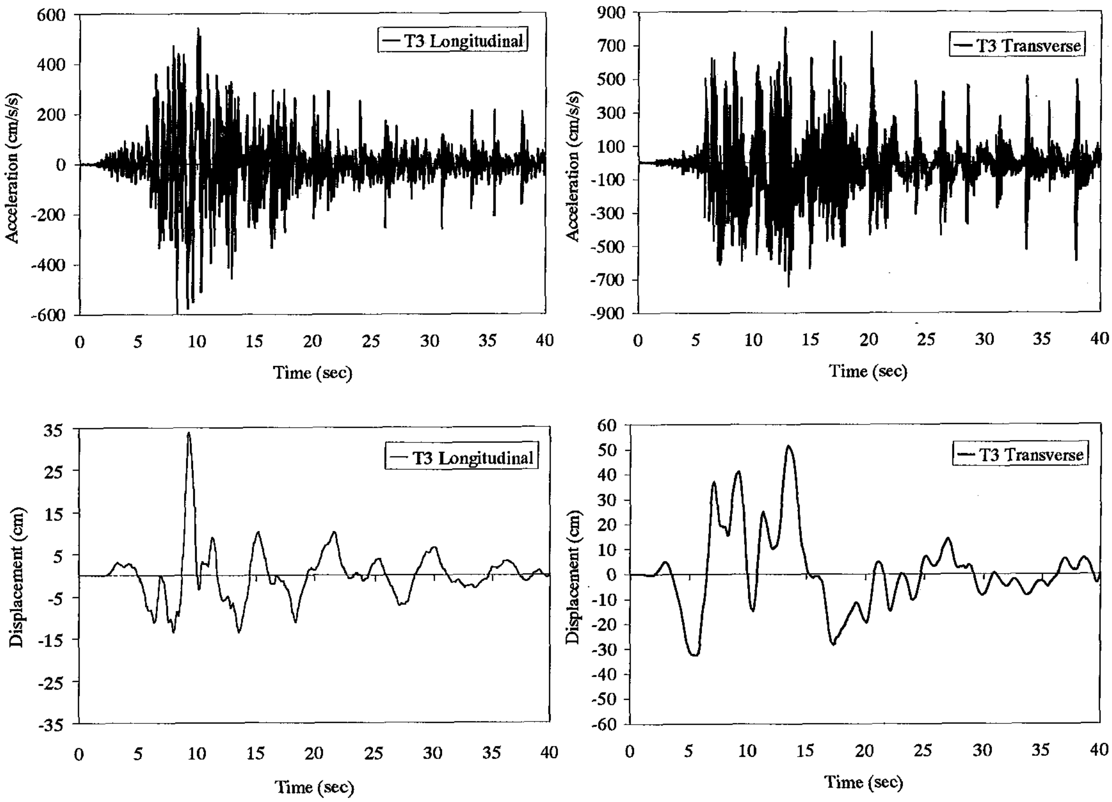


Fig. 15 Time histories of the Hyogoken-Nanbu Earthquake

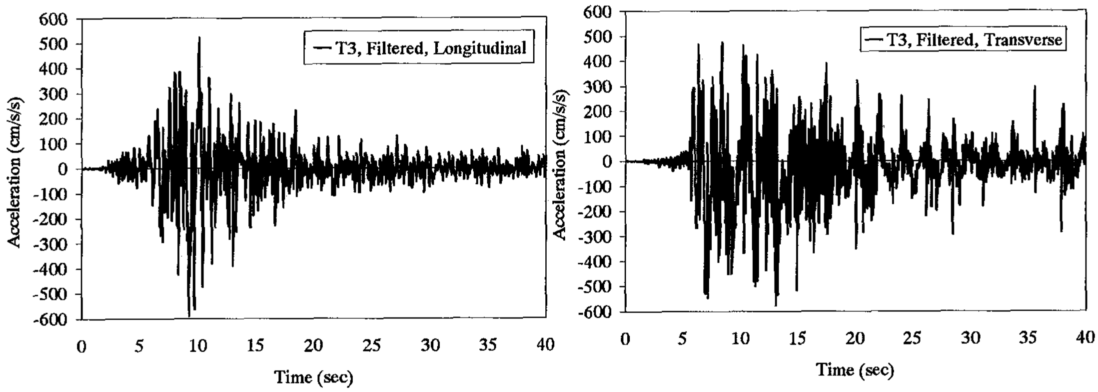


Fig. 16 Time history of acceleration from the Hyogoken-Nanbu Earthquake, filtered main girder level record

recorded time histories, and subsequently of the non-trivial damage occurred to the bridge.

Figure 15 shows the time histories of acceleration and displacement for the main girder level. Repeated pulses at approximately 2.5 seconds are observed. Moreover, some of the maximum amplitudes in acceleration time histories are coincident with the maximum amplitudes in displacement time histories. One can also observe that in the transverse direction the pulse is predominant and exhibits large amplitude. This suggests that the pulse in the longitudinal direction may be a secondary effect caused by the transverse motion. By stretching the acceleration record we found that the pulse is not formed of a single peak, like in pounding case, but many short period peaks. Thus, pounding of the bridge with the adjacent structure may not be the cause of this pulse phenomenon. This was suggested also by. The existence of similar pulses is observed also in the case of small earthquakes.

To remove short-period content from the acceleration record, a low-pass filter (0 to 12.5 Hz) was applied (Fig. 16). In this manner we can see that some of the pulses were removed from both the longitudinal and transverse recorded motions.

7. CONCLUSIONS

The response of the Higashi-Kobe Bridge during the Hyogoken-Nanbu Earthquake on January 17, 1995 was investigated. The scope of this analysis was to observe the complicated nonlinear soil and structure behavior. A comparison between the main shock and two after shocks was made.

The recorded response spectra were compared with the design response spectrum. All of the recorded response spectra surpassed the design spectrum in wide period ranges. This shows that the ground motion in the Hyogoken-Nanbu Earthquake was a very unfavorable one. Especially in the transverse direction can be observed that the ground motion was much stronger than expected.

A three-dimensional SASSI model was used. From the numerical analyses one can see that the model is able to simulate well the response during a small earthquake. For the Hyogoken-Nanbu Earthquake, the simulation is worse than in the case of the small earthquake. The complicated non-linear behavior of the bridge is emphasized in the Hyogoken-Nanbu Earthquake also because the bridge itself is a highly redundant structure. This shows that the model should be improved to simulate the response for the transverse direction.

A three-dimensional MSC/NASTRAN model was employed to find out that several cables were relaxed during the 1995 Hyogoken-Nanbu Earthquake.

From the investigation on non-linearity, it is possible that the pounding of the bridge with the adjacent structures did not cause the pulse phenomenon observed in the time histories. The problem being complex, further research is needed to find a realistic explanation.

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